Memorandum

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To: MR. AHMED IBRAHIM
Senior Bridge Engineer
Division of Engineering Services
Office of Bridge Design-West
Bridge Design Branch 4

Date: July 13, 2009

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Presidio Viaduct SB
Bridge No. 34-0157L

From: HOSSAIN SALIMI
Senior Materials and Research Engineer
Division of Engineering Services
Geotechnical Services - MS 5
Office of Geotechnical Design-West

Subject: Final Foundation Report

This report presents the Foundation Recommendations for the proposed 6-span Presidio Viaduct South Bound (34-0157L) in the city of San Francisco. This structure is part of the overall proposal to replace the existing 70-year-old Doyle Drive, a 1.5 mile (2.4 kilometer) stretch of Highway 101 in the northern part of the city extending from Marina Boulevard and Lombard Streets to the southern approach of the Golden Gate Bridge Toll Plaza. Doyle Drive lies entirely within the Presidio of San Francisco, which is presently part of the Golden Gate National Recreation Area, located within the Presidio Trust managed lands. Currently, Doyle Drive has non-standard design elements, including travel lanes from 9.5 to 10 feet in width, no fixed median barriers, and no shoulders.

It should be noted that a previous Foundation Report dated May 21, 2009 was submitted to you. However, the changes in the foundation/structure design as well as new construction requirements warranted changes to the foundation report. This report supersedes the recommendations contained in the May 21, 2009 report.

The new alignment includes over a dozen new structures, two tunnels, as well as retaining walls. It should be noted that the design of the project is being carried out as a partnership between Caltrans and Arup/PB consulting firms. The foundation reports for the structures and retaining walls will be prepared by Caltrans personnel. There will be separate reports for each structure forthcoming to your Office. This report will only cover the proposed 6-span Presidio Viaduct South Bound.

The following recommendations are based on the subsurface information gathered during the recent foundation investigations in 2008 and 2009 by the Office of Geotechnical Design-West (OGD-West) and Arup personnel, as well as General Plans/Foundation Plans provided by the Office of Bridge Design-West (OBD-West).

Project Description

The proposed bridge site is located in the city of San Francisco, less than a mile southeast of the Golden Gate Bridge Toll Plaza on Highway 101. The new Cast-in-Place Prestressed concrete structure is to replace the existing 18-span steel truss High Viaduct built in 1936. It should also noted that the design of the superstructure/bent locations was radically changed in March 2009 due to aesthetic and architectural requirement by the Presidio Trust, requiring additional geotechnical investigation by the OGD-West.

Geology

The OGD-West personnel performed two sets of foundation investigations for both the South-Bound and North-Bound structures. The first phase based on the General Plans and bent/column locations provided by the OBD-West in 2008, was performed between February and July 2008, and consisted of 24 mud-rotary borings drilled near the support locations using Christensen CS 1000, CS2000, Central Mine Equipment (CME) 75, CME 750, Mobile B-47, as well as Fraste Multi Drill XL and D51 drill rigs. Due to the aforementioned requirements by the Presidio Trust, and subsequent Bent/Column location changes, a second phase of investigation was needed. The second phase, based on the latest General Plans and bent/column locations provided by the OBD-West on March 5, 2009 was performed between late March and early April 2009. This investigation consisted of 8 mud-rotary borings drilled near the new support locations for both the proposed North-Bound and South-Bound structures using Christensen CS 2000, Fraste Multi Drill XL, as well as Acker MP-8 and MPCA drill rigs.

The regional geology can be traced back to the creation of the San Francisco Bay Area in the mid to late Pleistocene era. Like many areas in California, a long record of seismic activities characterizes the geological history of the Bay Area. In addition, the area has been strongly influenced by changes related to the Pleistocene glaciers and the presence of the San Francisco Bay Trough, resulting in variable thicknesses of recent deposits of soft to medium stiff Holocene clays (Young Bay Mud), an older, stiffer Pleistocene clay (Old Bay Mud) and sand deposits.

The project site lies with the Coast Ranges Geomorphic province. Cretaceous aged Franciscan formation rocks underlie the sediments. The Viaduct traverses a sediment filled topographic valley eroded into the top of the Franciscan formation. Rock is overlain by a relatively thin cover of soil at Abutment 1, Bents 5 and 6, and Abutment 7. The soil cover is at maximum at Bent 3.

For specific details regarding thicknesses and descriptions, as well as the formation descriptions, please refer to the Log-of-Test-Boring (LOTB) sheets for the proposed new bridge. The rock lithologies and conditions are shown on the LOTB sheets. It should be noted that many rock types were encountered including Serpentine, Shale, Sandstone, and Basalt. The rock strengths are highly variable, as are the degrees of weathering and fracturing encountered in the cores. For specific details regarding thickness and descriptions, as well as the formation descriptions, please refer to the LOTB sheets for the proposed new bridge.

Ground Water

During the 2008 subsurface investigation, piezometers were installed at three locations (Boreholes 34-0157NB-B3R, 34-0157NB-B4R, and 34-0157SB-B3L) along the Presidio Viaduct. Based on the latest readings dated July and October 2008, the water table is approximately 20 feet below ground near North-Bound Bent 3, 10 feet below ground near South-Bound Bent 3, and 16 feet below ground near North-Bound Bent 4.

Measured ground water elevations are also shown on the LOTB sheets. Ground water levels indicated in this report and shown on the LOTB sheets reflect the measured ground water level in the borehole on the specified date. Ground water surface elevations are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal conditions.

Scour Potential

The structure does not span watercourse. Therefore, there is no scour potential at the site.

Liquefaction Potential

As previously stated, the latest piezometer readings from the nearby borings show the water table to be approximately 10 feet below ground surface near Bent 3 and approximately 16 feet below ground near Bent 4. Due to the presence of high water table and some loose to medium dense granular materials, there is a moderate to high liquefaction potential from elevation –12 feet to –24 feet (depth 66 to 54 feet) at **Bent 3 Left**, moderate potential from elevation 0 feet to –5 feet (depth 45 to 40 feet), and elevation -34 feet to –39 feet (depth 74 to 79 feet) at **Bent 3 Right,** and moderate potential from elevation 17 feet to 11 feet (depth 36 to 30 feet) at **Bent 4 Right**. The liquefaction potential at the other support locations is very low.

Corrosion

A total of 41corrosion test results for soil samples were collected and submitted for testing from borings at Abutments 1, and 7, and Bents 2, 3, 4, and 5. The Corrosion Test Summary Report submitted by the Corrosion Technology Branch dated 12/31/08 is shown below in Table 1. The control corrosion parameters for this specific site are 3.2 pH, 340 ppm Chloride, and 13,160 ppm Sulfate. One sample at the South-Bound Bent 3 (90'-91.5') showed elevated Sulfite Content values.

Table 1 – Corrosion Test Summary

Depth and Location	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (ppm)	Chloride Content (ppm)
5.0-6.5 FT/NB-B3L	7.73	2703		
15.0-16.5 FT/NB-B3L	7.56	10647		
30.0-31.5 FT/NB-B3L	6.10	3919		
65.0-66.5 FT/NB-B3L	7.74	847	187	28
75.0-76.5 FT/NB-B3L	3.20	249	13160	50
95-96.5 FT/NB-B3L	7.69	847	350	63
10-11.5 FT/SB-B3R	7.11	16234		
20-21.5 FT/SB-B3R	7.27	13475		
30-31.5 FT/SB-B3R	6.70	3690		
40-41.5 FT/SB-B3R	6.70	3314		
55-56.5 FT/SB-B3R	6.62	3477		
75-76.5 FT/SB-B3R	7.69	600	73	73
90.0-91.5 FT/SB-B3R	4.86	598	2500	28
10-11.5 FT/SB-B2L	7.08	13915		
20-21.5 FT/SB-B2L	6.25	9204		

Table 1 – Corrosion Test Summary (continued)

Depth and Location	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (ppm)	Chloride Content (ppm)
35-36.5 FT/SB-B2L	6.92	9086		
50-51.5 FT/SB-B2L	7.31	2580		
10-11.5 FT/SB-B3L	6.77	10083		
20-21.5 FT/SB-B3L	6.88	8373		
25-26.5 FT/SB-B3L	5.85	3585		
35-36.5 FT/SB-B3L	6.88	3451		
55-56.5 FT/SB-B3L	7.64	2066		
80-81.5 FT/SB-B3L	6.59	1386		
125-126.5 FT/SB-B3L	7.39	1955		
5-6.5 FT/SB-B2R	8.06	11996		
15-16.5 FT/SB-B2R	7.31	14313		
30-31.5 FT/SB-B2R	7.39	10049		
5-6.5 FT/SB-B4R	6.53	7556		
15-16.5 FT/SB-B4R	6.83	6359		
30-31.5 FT/SB-B4R	5.71	1229		
70-71.5 FT/SB-B4R	7.10	1808		
5-6.5 FT/SB-B4L	7.24	7326		
15-16.5 FT/SB-B4L	6.95	5170		
30-31.5 FT/SB-B4L	7.09	6585		

Table 1 – Corrosion Test Summary (continued)

Depth and Location	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (ppm)	Chloride Content (ppm)
50-51.5 FT/SB-B4L	7.97	4950		
15-16.5 FT/SB-A1	7.41	4171		
5-6.5 FT/SB-B5R	7.47	3680		
11.5-13 FT/SB-A7	7.59	888	77	340
70-71.5 FT/SB-B4L	7.35	1333		
5-6.5 FT/SB-A1	6.32	2886		
5-10 FT/SB-A7	7.14	4135		

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Environmental/Hazardous Considerations

District 4 Division of Environmental Planning & Engineering completed an extensive soil investigation for the Doyle Drive project. The draft report titled "Soil Investigation Report" is in the process of being finalized and will be available as part of the supplemental handout for the contractor. The report will include a characterization of the soil and the groundwater of the entire Doyle Drive project for potential contaminants and hazardous waste. This includes testing and characterization of project areas subject to both roadway and structure excavation including CIDH pile locations, and the testing of all existing structures within the project for asbestos and lead-based paint.

For further information, please contact Mr. Ray Boyer, District Branch Chief - Hazardous Waste, Office of Environmental Engineering, Division of Environmental Planning & Engineering, Caltrans - District 04, at (510) 286-5668.

Fault and Seismic Data

The 2008 California Seismic Hazard Map (CSHM) derived from the latest United States Geological Survey (USGS) and California Geological Survey (CGS) maps shows the San Andreas fault zone (Strike-slip) with a maximum moment magnitude, Mmax=7.9 located about 9.5 kilometers southwest of the site, and the Hayward fault zone (strike-slip) with a maximum moment magnitude, Mmax=7.3 located about 20 kilometers northeast of the site. The controlling fault for the project site is San Andreas fault zone.

Based on the deterministic Seismic Hazard Analysis (DSHA) and using the Next Generation Attenuation Relationship equations incorporated into the CSHM, the peak bedrock acceleration (PBA) is 0.35g. However, based on the Probabilistic Seismic Hazard Analysis (PSHA), the PBA at this site, for a 975-year return period (5% probability of exceedance in 50 years) is 0.6g. Therefore, the PSHA governs the site. There are no known faults projecting towards or passing directly through the project site. Therefore, the potential for surface rupture at the site due to fault movement is considered low.

The OGD-West had submitted preliminary Acceleration Response Spectra (ARS) to your office in a memorandum dated April 13, 2006. Furthermore, a site-specific probabilistic seismic hazard analysis was performed to evaluate the hazard spectra for the suggested 75 year life span of the structures, with 20%, 15%, 10%, 5%, and 2% probability of exceedance, corresponding to return periods of 336, 460, 710, 1460, and 3700 years, respectively.

Attached please find bent-specific Recommended Design ARS curves for Functionality Evaluation Earthquake (FEE), which are based on a 108-year return period (50% probability of exceedance in 75 years) and Safety Evaluation Earthquake (SEE), which are based on a 975-year return period (5% probability of exceedance in 50 years). These curves were generated from the latest USGS and Caltrans on-line hazard maps.

Seismic Hazard Analysis

Due to the sensitivity of the project and the fact that the structure is considered a Recovery Route, more detailed geotechnical seismic analyses were required. The Joint Venture of Arup/ Parsons Brinkerhoff retained the services of Dr. Norm Abrahamson in 2008 to produce input rock motions for the needed analyses.

Dr. Abrahamson furnished the Department with three sets of spectrum compatible earthquake records with three components each (fault parallel, fault normal, and vertical) for both FEE and SEE. The consulting firm of Earth Mechanics, Inc. (EMI) was retained by the joint venture to produce column-specific free field motions for each of the supports at the proposed Presidio Viaduct using the input rock motions generated by Dr. Abrahamson. The soil profiles and parameters for each column location as well as P and S wave velocity values derived from geophysical loggings into bedrock strata at 6 locations through both North-Bound and South-Bound alignments were provided by this Office to EMI in March/April 2009.

In addition, the Lateral, Axial, as well as the Base Resistance Soil Springs (PY, TZ, and QZ curves) for each support location were provided to you by our Office in March/April 2009.

The final seismic data to include the column-specific time histories and recommended design response spectra will be provided by EMI.

Drivability Study

A drivability analysis was performed by the Geotechnical Services Foundation Testing Branch and a report dated May 14, 2009 was generated. This report details the drivability of a 144-inch steel casing at the proposed Bent 3R location using three different hammer models. For specific information, please refer to the attached report.

Permeability Study

A packer test was performed by ARUP at borehole 34-057SB-B2A near the proposed Bent 2 and a report dated May 2009 was generated. This report presents the results of the packer tests performed at this location by pumping water under sustained pressure, and measuring the rate of water flow into the rock formation. For specific information, please refer to the attached report.

Foundation Recommendations

According to the data provided by you in the latest General Plans dated 03/05/09, and subsequent transmittals, the foundation for the proposed replacement structure consists of Cast-In-Drilled-Hole (CIDH) piles supporting pile caps at the abutments and, single large diameter CIDH pile extensions at bents 2, 3, 4, 5, and 6. The pile sizes at the bents, as well as pile cap dimensions, and bottom of pile cap elevations at the abutments were provided by your Office in March 2009.

Abutment 1 consists of a 44.5 foot by 14 foot pile cap supported by twenty-six 24-inch diameter CIDH piles. The foundation at Bents 2, 3, and 4 consist of 12-foot diameter CIDH piles with 12-foot permanent steel casings, and 11.5-foot diameter CIDH rock sockets. The foundation at Bents 5 and 6 consist of single 11.5-foot and 8.5-foot CIDH piles, respectively, with column isolation from top of ground and extending to the pile cutoff elevations within the formational rock. Abutment 7 consists of a 51 foot by 14 foot pile cap supported by twenty-three 24-inch CIDH piles. The following tables provide the foundation details.

Table 2 - General Foundation Information Provided by Structure Designer

Support Location	Pile Type	Original Ground Elevation (Ft)	Pile Cut-off Elevation (Ft)	Permissible Settlement Under Service Load
Abutment 1	24-inch CIDH	81	71.25	25.4 mm (1 in)
Bent 2	144-inch CIDH with 138-inch CIDH rock rocket	58	36	25.4 mm (1 in)
Bent 3R	144-inch CIDH with 138-inch CIDH rock rocket	40	37.6	25.4 mm (1 in)
Bent 3L	144-inch CIDH with 138-inch CIDH rock rocket	42	37.6	25.4 mm (1 in)
Bent 4R	144-inch CIDH with 138-inch CIDH rock rocket	47	45	25.4 mm (1 in)
Bent 4L	144-inch CIDH with 138-inch CIDH rock rocket	51	45	25.4 mm (1 in)
Bent 5	138-inch CIDH	76	66	25.4 mm (1 in)
Bent 6	102-inch CIDH	121	101	25.4 mm (1 in)
Abutment 7	24-inch CIDH	136	129.5	25.4 mm (1 in)

Table 3 - Foundation Design Loads Provided by Structure Designer

Support Location	Service 1 Limit State (kips)			Strength Limit State (Controlling Group) (kips)				Extreme Event Limit State (Controlling Group) (Kips)			
	Total Loads		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Column	Max Per Pile	Per Support per column	Per Column	Max Per Pile	Per column	Max Per Pile	Per Column	Max Per Pile	Per Column	Max Per Pile
Abut 1	3429/supp	165	2965/supp	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	N/A	8310	7260	N/A	12440	0	0	N/A	7260	0	0
Bent 3R	N/A	5160	4355	N/A	8005	0	0	N/A	11525	N/A	2815
Bent 3L	N/A	5160	4355	N/A	8005	0	0	N/A	11525	N/A	2815
Bent 4R	N/A	6460	5385	N/A	10105	0	0	N/A	9518	0	0
Bent 4L	N/A	6460	5385	N/A	10105	0	0	N/A	9518	0	0
Bent 5	N/A	6420	5390	N/A	10020	0	0	N/A	5390	0	0
Bent 6	N/A	4200	3280	N/A	6810	0	0	N/A	3280	0	0
Abut 7	2679/Supp	174	2364/supp	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Foundation Design Recommendations for the abutments and bent locations are provided in the following three tables. Nominal Resistances are provided by a combination of side resistance and base resistance in rock. The Nominal Resistance for drilled shafts does not include any contributions from side shear in soil.

Table 4 - Foundation Design Recommendations for Abutments 1 and 7

Abutment Foundations Design Recommendations								
Support Location	Pile Type	Cut-off Elevation (Ft)	LRFD Service-I Limit State Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elevations (Ft)	Specified Tip Elevation (Ft)
			Total	Permanent				
Abut. 1	24-inch CIDH	71.25	3429	2965	165	330	36 (a)	36
Abut. 7	24-inch CIDH	129.5	2679	2364	145	290	90 (a)	90

Table 5 - Foundation Design Recommendations for Bent 2 through Bent 6

Support Location	Pile Type And size	Cut-Off Elevation (Ft)	Service-I Limit State Load per Column (Kips)	Total Permissible Support Settlement	Required factored Nominal Resistance (kips)				Maximum Specified Tip Elevation (Casing) (Ft)	Design Pile Tip Elevation (Ft)	Specified Pile Tip Elevation (Ft)
					Strength Limit		Extreme Event				
					Comp. (φ=0.6)	Tension (φ=0.6)	Comp. (φ=1)	Tension (φ=1)			
Bent 2	144-inch CIDH with 138-inch CIDH rock rocket	36	8310	1 inch	12440	0	7260	0	-9	-54 (a)	-54
Bent 3R	144-inch CIDH with 138-inch CIDH rock rocket	37.6	5160	1 inch	8005	0	11525	-2815	-121	-156 (a) -118 (b)	-156
Bent 3L	144-inch CIDH with 138-inch CIDH rock rocket	37.6	5160	1 inch	8005	0	11525	-2815	-103	-146 (a) -108 (b)	-146
Bent 4R	144-inch CIDH with 138-inch CIDH rock rocket	45	6460	1 inch	10105	0	9518	0	-30	-70(a)	-70
Bent 4L	144-inch CIDH with 138-inch CIDH rock rocket	45	6460	1 inch	10105	0	9518	0	-21	-60 (a)	-60
Bent 5	13[illegible]nch CIDH	66	6420	1 inch	10020	0	5390	0	N/A	25 (a)	25
Bent 6	102-inch CIDH	101	4200	1 inch	6810	0	3280	0	N/A	56 (a)	56

Notes:
1) *The design tip elevations at the abutments are controlled by (a) Compression.*
2) *The design tip elevation for settlement is not applicable at the abutments since the CIDH piles are required to penetrate into rock as reflected by the specified tip elevations.*
3) *CIDH specified pile tip elevations shall not be raised.*
4) *The design tip elevations for Lateral Loads are typically provided by Structure Design.*
5) *The design tip elevations for the Bents are controlled by: (a) Compression (b) Tension.*
6) *The specified tip elevation shall not be raised above the design tip elevation.*
7) *The design tip elevation for settlement is not applicable at the bents since the CIDH piles are required to penetrate into rock as reflected by the specified tip elevations.*

Table 7 - Pile Data Table

Location	Pile Type And size	Nominal Resistance		Maximum Permanent Casing Tip Elevation (Ft)	Design Tip Elevation (Ft)	Specified Tip Elevation (Ft)
		Compression (Kips)	Tension (Kips)			
Abut 1	24-inch CIDH	330	N/A	N/A	36 (a)	36
Bent 2	144-inch CIDH with 138-inch CIDH rock rocket	20,740	0	-9	-54 (a)	-54
Bent 3R	144-inch CIDH with 138-inch CIDH rock rocket	13,350	0	-121	-156 (a) -118 (b)	-156
Bent 3L	144-inch CIDH with 138-inch CIDH rock rocket	13,350	0	-103	-146 (a) -108 (b)	-146
Bent 4R	144-inch CIDH with 138-inch CIDH rock rocket	16,850	0	-30	-70 (a)	-70
Bent 4L	144-inch CIDH with 138-inch CIDH rock rocket	16,850	0	-21	-60 (a)	-60
Bent 5	138-inch CIDH	16,700	0	N/A	25 (a)	25
Bent 6	102-inch CIDH	11,350	0	N/A	56 (a)	56
Abut 7	24-inch CIDH	290	N/A	N/A	90 (a)	90

General Notes:

1) All support locations are to be plotted in plan view on the Log of Test Borings as stated in "Memo to Designers" 4-2. The plotting of support locations should be made prior to requesting a final foundation review.

2) When applicable, the structure engineer shall show on the plans, in the pile data table, the design pile tip elevation required to meet the lateral load demands. If the design pile tip elevation required to meet lateral load demands exceeds the specified pile tip elevations given within this report, the Office of Geotechnical Design-West shall be contacted for further recommendations.

Construction Considerations:

Cores Samples

1) Core samples from the 2008 and 2009 Caltrans foundation investigations are available for viewing by bidders at the California Department of Transportation, Transportation Laboratory, 325 San Bruno Avenue, San Francisco, CA 94103. The bidders are to allow the State five (5) working days to prepare and display the cores.

2) During the 2008 subsurface investigation, core samples were collected from several borings at 1.52 m (5 ft) intervals, when possible, and were submitted to the laboratory for strength testing. Some of the samples were so weak that they were unable to be tested. Per Standard Specifications, laboratory strength test data are available for viewing at California Department of Transportation, Transportation Laboratory, 5900 Folsom Boulevard, Sacramento, CA.

CIDH Piles

1) Ground water was encountered during the 2008 subsurface investigation and it is anticipated that the contractor will encounter ground water during CIDH pile construction. The static ground water levels indicated on the LOTB sheets reflect the measured ground water levels at the time of the piezometer readings. At the time of construction, the ground water surface elevations may be significantly higher or lower than those shown on the LOTB due to seasonal conditions, or the amount of water flowing in the region.

2) End bearing contributes to the nominal resistance of the CIDH piles at the bent locations. The contractor shall employ appropriate techniques to assure that the interface between the concrete and the rock at the base is free of drilling debris, so that the shaft concrete bears directly on the undisturbed rock. Caltrans field inspectors

shall inspect the base immediately prior to the concrete placement. The Caltrans inspection camera shall be used for inspection and the procedure shall be cited in the specifications.

3) The formational rock described as Shale, Sandstone, Serpentine, and Greywacke is extremely variable in hardness, fracturing, and weathering, ranging from very hard to very soft, slightly to very intensely fractured, and slightly weathered to decomposed. Therefore, at all support locations, the contractor should anticipate variable drilling conditions in the formational rock similar to the exploratory drilling conditions. The contractor should also anticipate the need to alternate from soft and hard rock drilling techniques to extend the drilled holes for the CIDH piles to the specified pile tip elevations.

4) During the 2008 and 2009 subsurface investigation, significant variations in the weathering, fracturing, and hardness of the sedimentary formational bedrock material, occurring within relatively short distances both laterally and vertically, were observed, and are shown in the LOTB sheets. In the formational units, the contractor should anticipate varying rock drilling conditions (alternating soft and hard rock drilling) across all the bent locations. The variations in rock conditions (described above) can occur from one pile location to the next pile location. The contractor should also be prepared for potential caving conditions within the formational unit. The amount of difficulty the contractor will experience will be dependent upon the methods and means the contractor chooses to use to construct the CIDH piles.

5) The contractor will need to use care while drilling the shafts for the CIDH piles. Due to the nature of portions of the formational rock units, rapid insertion and removal of the drilling tools during the drilling process can cause excessive scouring and caving of the walls of the drilled shaft.

6) The skin friction zones needed to calculate the geotechnical capacity of the CIDH piles used as well as the specified minimum casing top and bottom elevations for each pile are summarized below in Table 8. Casings are specified for one or several of the following reasons. Prevention of ground loss in the lowest strength foundation soils, and providing construction access to the shaft cut-off elevation.

7) The pile casing elevations for Bents 2, 3, and 4 are shown in Table 8. In addition, the column isolation for bents 5 and 6 are shown in Table 9. The column isolation is to provide a gap between the column above the cut-off elevation and the foundation rock. The casing bottom elevations shown below are the highest elevations allowed. The contractor may choose construction equipment and construction techniques that require casing diameters, wall thicknesses, casing types, and casing tip elevations which are different than those shown in the plans and specifications. It is expected

that the drilling contractor will use his expertise to provide a casing configuration that meets his needs.

8) The contractor may choose methods other than casing below the specified permanent casing elevations to stabilize the drilled holes.

Table 8 - CIDH Pile Casing lengths and Skin Friction Zone Elevations

Location	Ground elevation O.G (Ft)	Casing type and size O.D. (Ft)	Pile cutoff elevation (Ft)	Maximum Bottom of casing elevation (Ft)	Skin Friction Zone Start Elevation (Ft)	Skin Friction Zone End Elevation (Ft)
Bent 2	58	12	36	-9	-9	-54
Bent 3R	40	12	38	-121	-121	-156
Bent 3L	42	12	38	-103	-103	-146
Bent 4R	47	12	45	-30	-30	-70
Bent 4L	51	12	45	-21	-21	-60

Table 9 - CIDH Column Isolation and Skin Friction Zone Elevations

Location	Ground elevation O.G (Ft)	Pile cutoff elevation (Ft)	Top of column isolation casing elevation (Ft)	Bottom of column isolation casing elevation (Ft)	Skin Friction Zone Start Elevation (Ft)	Skin Friction Zone End Elevation (Ft)
Bent 2	58	36	58	36	-9	-54
Bent 5	76	66	76	66	66	25
Bent 6	121	101	121	101	101	56

Permanent and Temporary Steel Casings

The contractor should anticipate numerous difficulties while installing the permanent and temporary steel casings:

1) The recommended permanent steel casings, at Bents 2, 3R, 3L, 4R, and 4L locations, are intended to minimize construction difficulties due to caving of the loose alluvial material overlying the sedimentary formational material. The permanent steel casings will not eliminate the potential for caving within the formational material below the casing tip elevation. The methods and means used by the contractor to install the permanent steel casings and seal the contact between the casing tips and formational material, will directly determine the construction difficulties the contractor will encounter with the overlying alluvium while excavating the CIDH pile in the underlying formational materials.

2) If telescoping temporary casing is used in the formational layer(s) to control caving, the contractor should size the permanent steel casing accordingly to prevent damage to the permanent steel casing during CIDH pile excavation.

3) The contractor should not assume that installation of the permanent steel casing would allow the contractor to drill and place concrete for the CIDH piles in the dry. The intent of the permanent steel casing is only to minimize construction difficulties due to caving of the loose alluvial material overlying the formational material.

4) During construction of the CIDH piles, the contractor should choose appropriately sized drilling tools so as to make sure not to damage or puncture the wall of the permanent steel casing while inserting and removing the drilling tools. Puncturing the wall of the steel casing may allow alluvium to enter the drilled hole. Alluvium flowing through the punctured casing and into the borehole can potentially lead to subsidence at the ground surface.

5) During construction of the CIDH piles, the contractor should make sure to maintain a positive head between the ground water surface and the fluid level inside the permanent steel casing. If a positive head is not maintained, it can potentially lead to caving of the walls of the drilled hole, and construction difficulties.

6) At the Abutment, the contractor shall not have adjacent drilled shaft open simultaneously. The drilled shaft shall be drilled and filled with concrete before any adjacent holes are drilled. It should be noted that the CIDH holes at the abutments would only have a minimum of 24 inches distance from edge to edge. The requirement not to have adjacent drilled shaft holes open simultaneously is to prevent caving in or jeopardizing the integrity of the drilled holes.

The recommendations contained in this report are based on specific project information including structure type, support locations, and design loads which have been provided by the Office of Bridge Design-West. If any conceptual changes are made during final project design, the Office of Geotechnical Design-West should review those changes to determine if these foundation recommendations are still applicable.

If there are any questions, please contact Hossain Salimi at (916) 227-7147.

Attachments

cc: R.E. Pending File
John Stayton - Specs & Estimates (4)
District 4 (Project Manager)
Majid Madani
Tim Pokrywka – OGD-West
Project File

HOSSAIN SALIMI
No. 54567
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CIVIL